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Reference: 017052.100

July 20, 2017

Dan Berman, City Manager
City of Trinidad
P.O. Box 390
Trinidad, CA 95570

Subject: **Landslide Mitigation Assessment, Trinidad Memorial Lighthouse and Edwards Street, Trinidad, California**

Executive Summary

In light of the observed ground displacements over the previous two wet seasons, we are of the opinion that failure to stabilize at least a portion of the head of the landslide complex or move the lighthouse prior to the onset of the coming winter could result in the structure and foundation being comprised. Addressing the portion of the landslide with the potential to affect Edwards Street directly is less critical at this time, but should be considered in the coming years if the landslide continues to grow in the upslope direction.

The purpose of this initial phase of the investigation is to provide conceptual slope stabilization alternatives to protect Edwards Street and/or the lighthouse from the most threatening portion of the landslide complex. Stabilization of the entire landslide complex is not part of this project. Portions of the landslide that are located below any proposed soil reinforcement or retaining system will likely continue to move downslope after the slope repair is constructed.

An understanding of the local geology, failure mechanism, and landslide geometry is paramount for assessing the likely effectiveness of various slope stability mitigation measures. Stakeholders should be aware that the ground surface upslope of the existing landslide headscarp is likely to experience future displacement that will most certainly damage the lighthouse and, potentially, the roadway if no action is taken in the immediate to near future.

Based on the site conditions, the likelihood for continued landslide movement in the coming years, and the limitations of the various repair options described in the following document, we recommend that the lighthouse be relocated. With regard to Edwards Street, there is currently a buffer between the head of the slide and the edge of the roadway such that there is a lesser immediate risk factor as compared to the lighthouse. However, we expect the head of the landslide to continue encroaching toward Edwards Street within the next few years. We, therefore, recommend that the City of Trinidad strongly consider the construction of a retaining wall system to preserve the full traveled roadway width of Edwards Street, a main transportation artery in the town of Trinidad.

Introduction

This document presents SHN Engineers & Geologists' assessment of recent landslide activity along Edwards Street and adjacent to the Trinidad Memorial Lighthouse. This report provides conceptual slope repair/stabilization alternatives and recommendations regarding future actions. The main objective of this report is to provide the City of Trinidad and Trinidad Civic Club with an understanding of site conditions in order to inform their decision making process with regard to protecting Edwards Street and the lighthouse.

This investigation is the result of recent landsliding at the site, which has experienced a significant pulse of renewed movement over the past two rainy seasons. During the past winter season, landsliding at the site was responsible for the formation of fresh scarps and fissures in close proximity to the existing Lighthouse memorial and the parking area along the southern edge of Edwards Street just west of the lighthouse. The sliding resulted in significant damage to sidewalks and other hardscaped areas surrounding the lighthouse, and encroached on the southwest corner of the lighthouse foundation pad. The general site location is as shown on Figure 1. A depiction of the existing site conditions, including the limits of the landslide complex in relation to site features, is provided in Figure 2. The bluff profile included in Figure 2 shows our interpretation of the multiple failure surfaces actively deforming the project site.

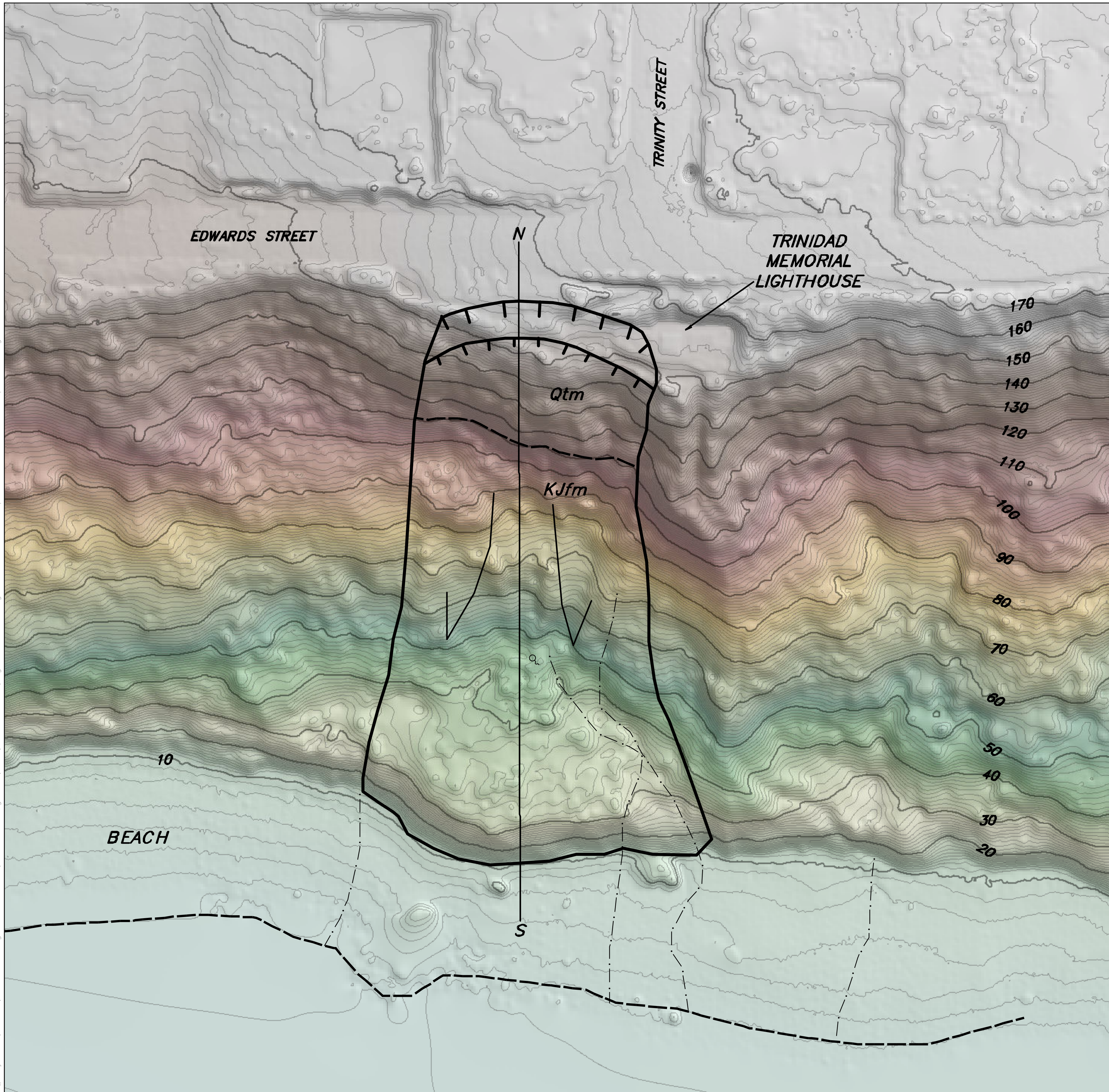
Work performed by SHN as part of this investigation includes field and aerial photographic mapping, conducting a subsurface drilling investigation, constructing and monitoring slope inclinometers, and quantitative slope stability modeling.

Slope stabilization alternatives presented in this report run the spectrum from soil reinforcement, including driven spiral soil nails and drilled soil nails, to a large-scale retaining system consisting of a soldier pile wall with tiebacks. Other pile walls (such as, cantilevered soldier piles and a tangent piled wall) were considered, but dismissed based on their high cost versus likely effectiveness. Current site conditions will pose challenges to the constructability for whatever stabilization method is chosen. These include the limited available space for staging and working areas, limited accessibility to the steep bluff face, the regulatory and permitting environment, and the historical and cultural sensitivity of the site.

Field Investigation and Monitoring Program

On May 3 and 4, 2017, four geotechnical borings, denoted as TML-1 through TML-4, were drilled at the locations shown on Figure 3. The borings were drilled to assess soil conditions, determine the depth to bedrock and the slide plane, and to make a direct measurement of the groundwater elevation at the time of drilling. The borings were drilled and sampled to depths of up to 51 feet below the existing ground surface (BGS) by Taber Drilling of West Sacramento, California. Borings were advanced using both a truck-mounted and limited-access track-mounted drilling rig outfitted with 6⅝-inch diameter hollow stem augers and 4-inch diameter solid flight augers.

SHN Consulting Engineers & Geologists, Inc.	City of Trinidad Landslide Mitigation Assessment Trinidad, California	Project Location SHN 017052.100
July 2017	Figure1_ProjectLocationMap	Figure 1



EXPLANATION

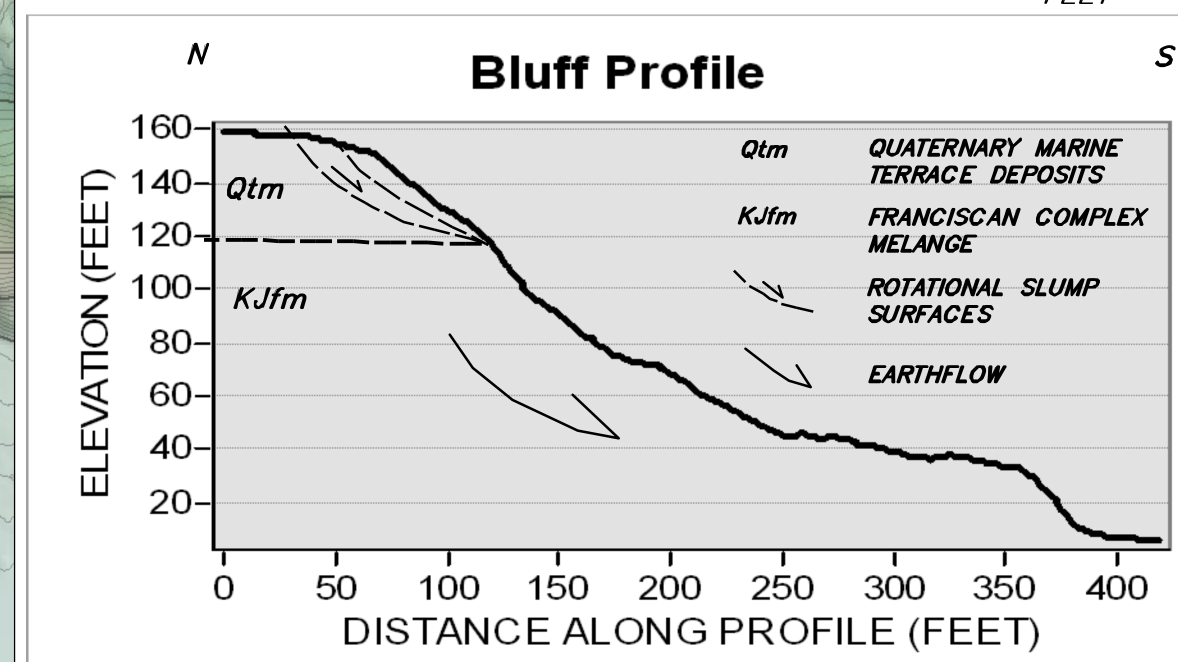
Qtm	QUATERNARY MARINE TERRACE DEPOSITS
KJfm	FRANCISCAN COMPLEX MELANGE
	LANDSLIDE HEADSCARP
	EARTHFLOW
N—S	BLUFF PROFILE LINE
160	TOPOGRAPHIC CONTOUR IN FEET
	TRINIDAD MEMORIAL LIGHTHOUSE
	STREAM
	SPRING

1"=70'±

0 70

FEET

NOTE: ELEVATION DATA FROM 2016 USGS WEST COAST EL-NINO LIDAR



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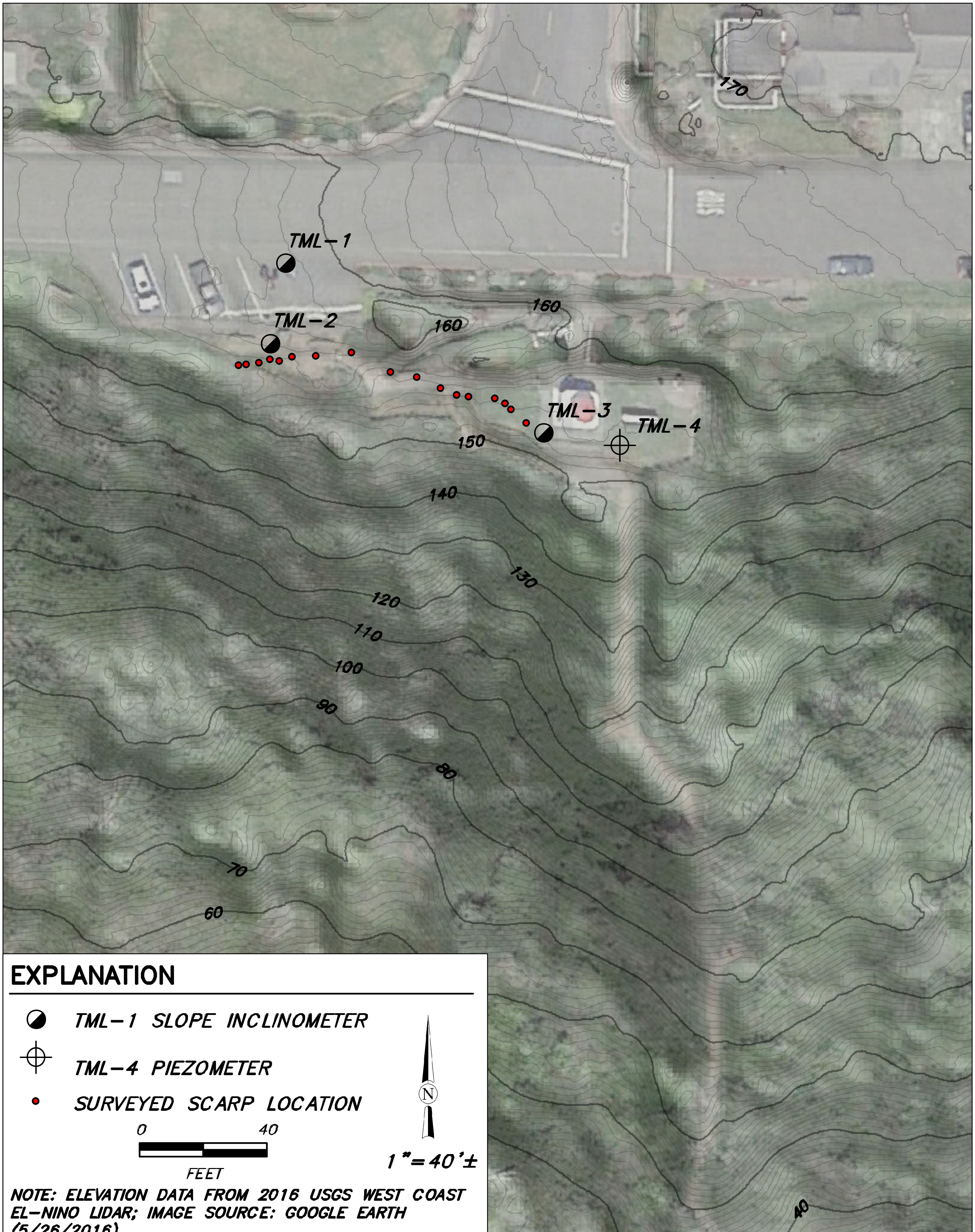
City of Trinidad
Landslide Mitigation Assessment
Trinidad, California

Existing Site Conditions Map
SHN 017052.100

July 2017 | Figure2_ExistingSiteConditionsMap | Figure 2



Path: \\eureka\projects\2017\017052-LighthouseGEO\100-DrillingGEO\GIS\PROJ_MXD\Figure3_BoringLocationMap.mxd User Name: psundberg

SHN Consulting Engineers & Geologists, Inc.	City of Trinidad Landslide Mitigation Assessment Trinidad, California	Boring Location Map SHN 017052.100
July 2017	Figure3_BoringLocationMap	Figure 3

During drilling, representative samples were obtained using modified California (2.5-inch internal diameter [I.D.]) and standard penetration test (SPT; 1.4-inch I.D.) split spoon samplers. SPT was performed to estimate the relative in-place density of the soils. Soil strength parameters of the cohesionless soils were estimated from correlations with SPT blow counts. Split-spoon samplers were driven by a 140-pound hammer dropping 30-inches inside the boring, controlled with an auto-hammer. The subsurface materials encountered were logged and field classified in general accordance with the Manual-Visual Classification Method (ASTM-International [ASTM] D 2488). Final boring logs are presented in Attachment 1 and were prepared based on the field logs and examination of samples in the laboratory.

The borings and pertinent site features were surveyed and tied to City of Trinidad monuments by SHN under the direction of a licensed surveyor. Surficial reference points were also established and surveyed to monitor future ground displacement. A survey point file with horizontal and vertical coordinates is included in Attachment 2.

Boring TML-1 was drilled at the outboard edge of Edwards Street and is located to the landward (upslope) side of the main head scarp and active slide mass. The boring was drilled to a depth of 50 feet BGS and was located primarily to assess the material strength of the undeformed terrace deposits. Boring TML-2 was drilled to a depth of 51 feet BGS immediately downslope of the main head scarp near the apex of the active slide mass. Boring TML-3 was drilled to a depth of 35 feet BGS and is located near the southwest corner of the concrete sidewalk surrounding the lighthouse. Slope inclinometer casing consisting of 2.75-inch outside diameter PVC was constructed in each of these boreholes to monitor landslide movements and determine the depth(s) to the slide planes. The slope inclinometer in TML-1 is intended to be used for future monitoring in order to assess whether or not the landslide is retrogressing upslope and beneath Edwards Street. One additional boring was drilled to a depth of 35 feet BGS east of the lighthouse. The boring was converted to a piezometer constructed of 1½-inch PVC pipe with factory-screened 0.010 slot in order to measure groundwater levels. All borings were completed with traffic-rated well boxes at the ground surface to protect the top of the casings and for ease of access for future monitoring.

Subsurface Conditions

Bedrock/Soil

Franciscan Complex bedrock was encountered at a depth of approximately 45 feet BGS and 42.5 feet BGS (elevation = 114± feet) in borings TML-1 and TML-2, respectively. Both borings are located along the approximate longitudinal axis of the landslide complex. Borings TML-3 and TML-4 were advanced to 35 feet BGS (elevation = 117± feet), but did not encounter bedrock. Where encountered, bedrock consists of dense to very dense graywacke sandstone that appeared to be a relatively fresh in hand specimen with moderate field strength. Based on the nature of mélange, natural exposures at the back edge of Old Home Beach (aka Indian Beach) and around Trinidad Bay, and the geometry of the slope profile, we expect bedrock underlying the site to consist of a mixture of hard rock blocks in a matrix of pervasively sheared, and moderately to highly decomposed argillite and shale with very weak field strength. The exposures at beach level generally indicate "bedrock" to consist of mélange matrix, which has the consistency of very stiff fine grained clay-rich soil with a block-in-matrix texture. Bedrock encountered in the borings likely constitutes a coherent rock block "floating" within the mélange matrix and should not be construed as being representative of the entire bedrock section.

Where mélange matrix is abundant, the mélange is highly susceptible to downslope mass movement in the form of earthflows. When saturated, the mélange matrix possesses very low internal strength and has the tendency to creep downslope at a very low angle of repose. The areas along Trinidad Bay are entirely underlain by Franciscan mélange with a block-in-matrix texture, which is responsible for the bay's unique shape. Stability of the overlying terrace sediments can be influenced by the presence of large sandstone or greenstone blocks in the underlying mélange. No such large, coherent rock block exists downslope of the project site. Therefore, the terrace sediments deform in direct response to movement within the underlying mélange and will likely continue to do so until a favorable slope angle of repose is achieved.

Overlying bedrock is Pleistocene age (approximately 60,000 to 100,000 years old) marine terrace sediments composed chiefly of fine to medium sand with scattered fine gravel. The terrace deposits are present beginning at the ground surface and continue to a depth of least 35 feet BGS to as much as 45 feet BGS. The terrace sediments have soil textures ranging from silty sand (SM) to poorly-graded sand (SP). The gravel content increases with depth where, presumably, a coarse gravel to cobble lag deposit lies directly on the bedrock surface. The terrace materials are weakly cemented and disaggregate, with little to no finger pressure; this indicates relatively low dry strength. The granular soils generally contain non-plastic fines with intermittent discontinuous lenses of fine rounded gravel. SPT blow counts indicate the relative density of the granular soils to be loose to medium dense throughout the soil profile. The relative density of the terrace materials increases slightly below a depth of about 25 to 30 feet. Practical refusal occurred at the terrace/bedrock contact.

Groundwater

The initial depth to groundwater at the time of drilling was observed in TML-1 and TML-2 at 35 feet BGS and 42 feet BGS, respectively. The noted groundwater depths are interpreted to reflect perching of the groundwater surface on top of the soil/bedrock contact. The relatively steep groundwater surface gradient is likely due to the proximity of the slope free face, whereby groundwater is allowed to emerge rapidly in the form of seeps and springs. A reconnaissance of the lower and mid-slope portions of the slide body as recently as mid-June revealed the presence of saturated surface conditions and flowing surface runoff, indicating that a high volume of emergent groundwater persists throughout the wet season and early dry season months. Due to the low transmissivity of the mélange bedrock, we interpret that most, if not all, of the surface runoff observed on the mid- and lower slopes emerges from the bluff face at the elevation of the soil/bedrock contact.

Site Observations and Interpretations

The lighthouse is sited on a cut bench constructed into the original hillslope. The site was originally constructed in the late 1940s (the cut bench is visible in the 1948 aerial photographs) and has remained relatively unaffected by localized landsliding until the previous two wet seasons. The parking area at the outboard edge of Edwards Street and the concrete walkways to the west of the lighthouse began to display ground displacement and distress following the onset of winter rains around early 2016. Vertical and lateral ground displacements continued through the 2016 winter and accelerated during the 2016/2017 winter.

Rainfall totals during 2015/2016 were considered normal to slightly above average compared to historical averages. Rainfall totals during 2016/2017, however, were well above historical average; rainfall has likely contributed significantly to the increased rate of slope movements observed at the site. Of particular noteworthiness is the fact that ground deformation in the form of a tension crack along a previously identified scarp has continued to occur well into the summer dry season.

The original cut pad on which the lighthouse was constructed appears to have extended much further west than its current position, suggesting that slope movement and bluff retreat have been occurring historically. Numerous large trees that were once visible on the slopes descending to the beach are no longer present and have since been replaced with low-lying shrubs; this may be attributed to recurrent slope movements. Not until the recent wet seasons has landsliding begun to directly affect the upper bluff edge in such a manner that the upper portion of the landslide complex has become readily visible.

The formation of multiple, nested, arcuate failure planes observed at the head of the slide complex indicates that the upper portion of the landslide is a large retrogressive rotational slide (it is progressing upslope and is failing along a main curved slip surface). It is unclear from a review of the historical aerial photographs dating to 1948 when landsliding initiated on the slopes below Edwards Street and the lighthouse. However, the growth of an outwardly convex bulge at the toe of the coastal bluff slope along the back edge of Old Home Beach is readily apparent beginning in the 1965 photographs. This landform is suggestive of an active deep-seated earthflow within the mélange bedrock that began prograding (advancing) seaward at that time. Landsliding on the midslope may have initiated soon after the initial deep-seated failure, but was likely obscured by the thick vegetation.

The upper limits of recent landsliding at the site are actively deforming the outer 25 feet of the bluff top along multiple slip surfaces. The main head scarp at the points where it intersects the bluff edge at the main break in slope measures up to 100 feet wide. Lateral margins farther downhill are not apparent; they are obscured by thick vegetation. We assume that the nested failure planes sole (detach at a low angle) into a single slide plane at depth. The slide plane is interpreted to be confined to the marine terrace sands that overlie the mélange bedrock and would, theoretically, daylight along the slope face about 45 to 50 feet in elevation below the bluff edge. The maximum width of the body of the entire landslide complex is approximately 140 feet midslope, and is assumed to extend a ground distance of about 400 feet down slope from the bluff edge, as shown on Figure 2.

Geotechnical Analysis

Downhole measurements collected from the slope inclinometers constructed in borings TML-1 through TML-3 have not shown any significant displacement to date due to the timing of their construction (after the end of the rainy season). Less than ¼-inch of deflection has been measured for TML-2 and TML-3; this is within the range of measuring bias for the slope indicator instrument used. Based on landslide geometry and slope profiles generated from the 2016 LiDAR data, we estimate that the depth of the slide planes is approximately 20 to 30 feet below the location of the outer bluff edge.

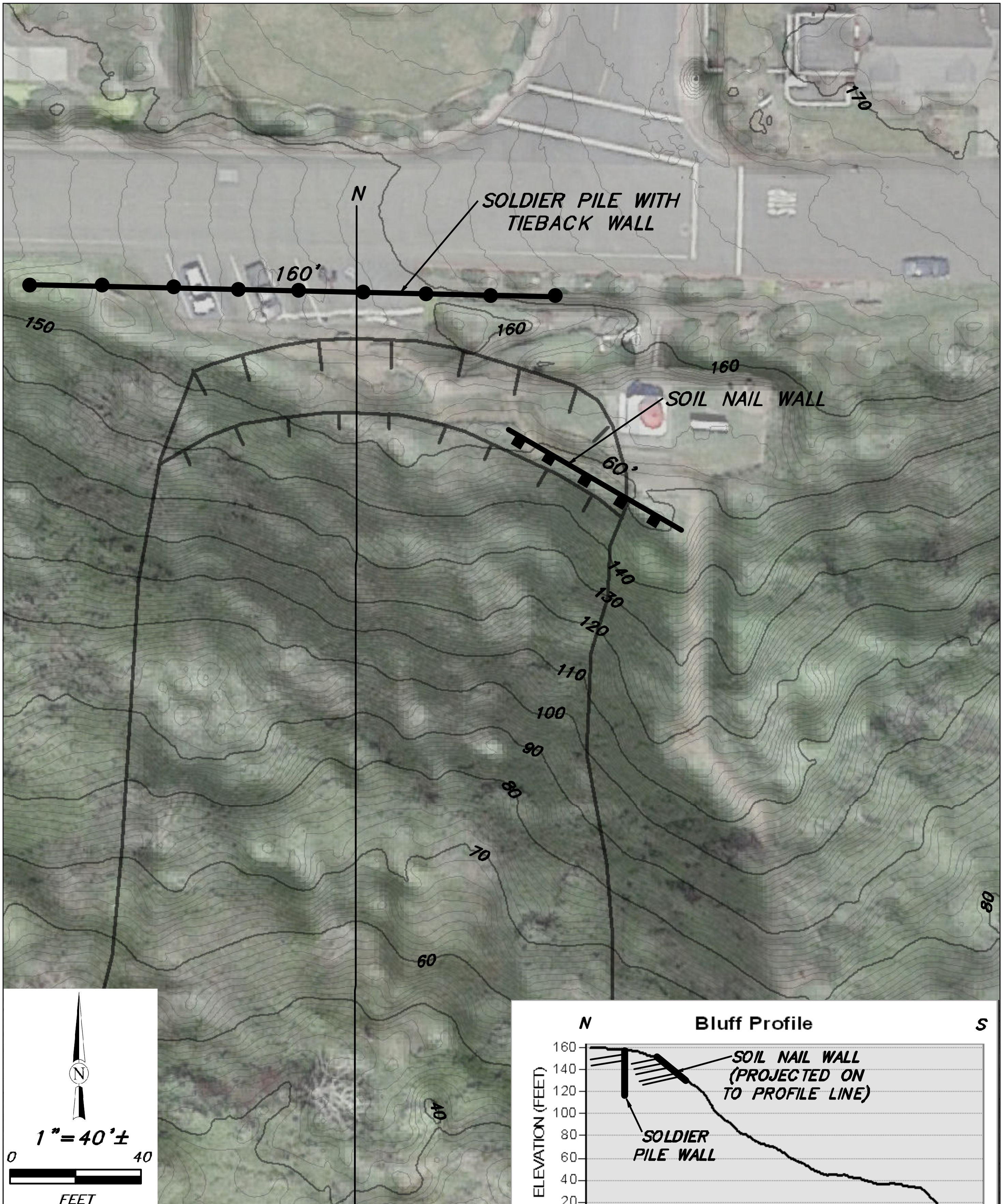
Slope stability analyses were performed using computer software (SLIDE 7.0) and the slope profile line depicted on Figure 2. The output for the model of the existing failure condition was determined using the Spencer method and a fully specified failure surface. (This is a simplified three-layer model in which soil and bedrock strength parameters were back calculated with an interpreted piezometric groundwater surface, to achieve a factor of safety of 1.0.) The assumed depth of the failure plane was estimated to be approximately 25 feet BGS at the location of the bluff edge. The effective geotechnical parameters for the slide materials are phi (angle of internal friction) equal to 28 and 30 degrees, cohesion equal to zero (0) pounds per square foot (psf), and saturated unit weight equal to 120 and 130 pounds per cubic foot (pcf). The shear strength of the bedrock was modeled as 5,000 psf with saturated unit weight equal to 144 pcf.

The ground anchor forces required to resist the landslide forces were then evaluated using driven spiral soil nails, drilled soil nails, and a soldier pile with tie-back wall.

Driven Spiral Soil Nails

The use of spiral nails in conjunction with reinforcing wall facing (such as, anchored mesh) is considered the most economical and feasible retaining system from a construction standpoint. The spiral nail system is a soil reinforcement method commonly employed for stabilizing steep cut slopes of limited height. They can be used with variety of slope face reinforcements, including wire facing (recommended due to strength to weight ratio) and gabion facing. The soil nails are driven into the slope face using a pneumatic hammer attached to the end of a long reach excavator. Areas to be treated with this method would need to be cleared of vegetation.

This type of retaining system would require the least amount of heavy equipment traffic and would minimize the amount of disturbance to the site during construction relative to the other proposed alternatives. This type of retaining system is also considered the least robust in terms of longevity and long-term effectiveness. This stabilization method would be suitable for reinforcing the slope face in the immediate vicinity of the lighthouse, only, and is not recommended for stabilizing the portion of landslide along Edwards Street. The location of this type of soil reinforcement system is shown on Figure 4.



NOTE: ELEVATION DATA FROM 2016 USGS WEST COAST EL-NINO LIDAR; IMAGE SOURCE: GOOGLE EARTH (5/26/2016)

SHN Consulting Engineers & Geologists, Inc.	City of Trinidad Landslide Mitigation Assessment Trinidad, California	Conceptual Retaining Wall Locations SHN 017052.100
	July 2017	Figure4_ConceptualRetainingWallLocation
		Figure 4

The advantages associated with spiral soil nailing include the following:

- Spiral soil nails would result in less long-term environmental impact compared to other construction techniques such as drilled shafts or soldier pile walls, which require relatively large equipment. Short-term impacts are related to vegetation clearing within the work area.
- The installation of spiral soil nails is relatively fast.
- Soil nailing may be more cost-effective at sites with remote access, because the smaller equipment is more readily mobilized.

The main disadvantage of this approach is the limitation to application in the upper part of the slope. It is not feasible to cover the entire slope, due to lack of access; it will only be feasible to install nails where the long-reach excavator can reach, which will limit the effectiveness of this method. There may also be issues associated with driving the spiral nail the full design length. The presence of medium dense granular soil (sand and gravel) may inhibit the installation of the spiral soil nail for its full length. Shorter nail lengths may prove ineffective in attaining the desired factor of safety against landsliding.

We anticipate constructing such a system for a horizontal length of about 60 feet and slope distance of about 40 feet beginning at the bluff edge (this is the inferred limit of the long-reach excavator) and down the slope face, for a total area of about 2,400 square-feet. For modeling purposes, reinforcement loads for the spiral soil nails were applied at 16 degrees from horizontal and spaced 7-feet apart laterally and 10-feet apart along the descending slope face. The reinforcement loads were determined by increasing the spacing in the stability analysis until the target factor of safety of greater than 1.3 was achieved. The results of the stability analysis indicate that an equivalent pull out bond strength of 940 lbs/ft for three rows of 30-foot spiral soil nails and three rows of 40-foot spiral soil nails is required for a 20-foot vertical design height. Greater factors of safety could be achieved by reducing the nail spacing and/or increasing the overall vertical height.

Drilled and Grouted Soil Nails

Soil nails are reinforcing, passive elements that are drilled and grouted sub-horizontally into the ground to support excavations in soil, or in soft and weathered rock. They contribute to the stability of earth-resisting systems mainly through tension as a result of the deformation of the retained soil or weathered rock mass. Tensile loads are transferred to the surrounding ground through shear stresses (bond stresses) along the grout-ground interface. This type of commonly used soil reinforcement method has long-term, demonstrable corrosion protection to ensure adequate, long-term performance of the system.

Soil nail walls are constructed using a "top-down" construction sequence, where the ground is excavated in lifts of limited height. The extent of excavation required at the project site would be limited to removing any loose soil material from the slope face to provide a smooth surface. Soil nails and an initial shotcrete facing are installed at each excavation lift to provide support.

Subsequently, a final shotcrete or cast-in-place-concrete (CIP) facing is installed. Nails are most often installed at a vertical spacing of 4 to 6 feet. The nail vertical spacing is comparable to the typical height of a stable, excavation lift, which is commonly 3 to 5 feet. The horizontal spacing of nails is often also in the range of 4 to 6 feet. Both the soil nails and the initial and final facing contribute to the stability of the excavation. The soil nails support the soil and transfer loads to the soil mass behind the wall. The facing supports the soil between nails and immediately behind the face, provides structural continuity, and enables the soil nail wall to act as a unit.
The advantages associated with drilled and grouted soil nailing include the following:

- Soil nail walls cause less long-term environmental impact compared to other construction techniques (such as, drilled shafts or soldier pile walls), which require relatively large equipment. Short-term impacts are related to vegetation clearing within the work area.
- The installation of soil nail walls is relatively fast.
- Easy adjustments to nail inclination and location can be made when obstructions are encountered (such as, boulders or underground utilities).
- Soil nailing may be more cost-effective at sites with remote access, because the smaller equipment is more readily mobilized.
- Soil nails are installed using equipment that is multipurpose and can be used for other substructure elements (such as, underpinning or protection of adjacent, movement-sensitive structures, like the lighthouse).
- Soil nail walls can accommodate curves and "bends" more easily than other top-down construction wall systems, which would otherwise require straight wall segments.

Similar to the spiral nail reinforcement system discussed previously, we anticipate constructing a drilled and grouted soil nail wall system to stabilize the slope immediately below the lighthouse only and at a similar location as shown on Figure 4. Similar to the spiral nail approach, the drilled nail approach is limited to treatment of the upper slope, which will limit its effectiveness. The dimensions of a drilled and grouted soil nail wall would, therefore, also be on the order about 2,400 square-feet.

Soldier Pile and Tie-back Wall

A retaining structure consisting of soldier piles, lagging, and one or two rows of tie-backs is feasible along the outboard edge of Edwards Street and at the former locations of the concrete walkways west of the lighthouse. Such a system would likely prove difficult to construct in the immediate vicinity of the lighthouse cut pad due to space limitations and equipment access limitations, unless a large access road was graded, and the lighthouse was temporarily relocated (or removed and reconstructed) during construction.

As conceptually proposed, the retaining structure would consist of deep soldier piles constructed of vertical steel elements in conjunction with a timber lagging wall, which would be partially restrained by one or more rows of tiebacks drilled into the hillside. The wall would be located as shown on Figure 4. The west end of the wall would terminate approximately 20 to 30 feet outside the limits of the portion of the landslide complex that threatens the upper bluff edge along Edwards Street for a total wall length of about 140 to 160 feet. At the wall location, the landslide slip surface is interpreted to be 10 to 20 feet in depth, and as much as 40 feet deep downslope of the wall location. It is likely that the existing landslide deposits (the failed portion of slope) will continue to translate downslope, therefore the wall should be designed to penetrate to the bedrock surface. Therefore the soldier piling will approach 40 to 50 feet in depth to penetrate a sufficient distance below the failure surface along the entire length of wall.

The intent of the soldier pile wall directly above the landslide is to create a barrier to isolate Edwards Street from the slide. We assume that the landslide mass below the wall would continue to move and eventually pull away from the downhill side of the wall. This could result in the loss of material between the pilings. It would be impractical to construct lagging for the full height of the wall due to the amount of excavation of the slope face that would be required. Therefore, it is recommended that soldier piling be spaced at three pile diameters or closer (center to center) such that soil arching would occur between the pilings to limit the internal loss of soil without the use of lagging to the full depth. Lagging should be installed at the top of wall and extend at least 5 feet below the lowest tie-back level. One or more rows of anchored tie-backs would be required from the face of the wall to a point that is several tens of feet (approximately 40 feet or more) beyond the existing and expected future critical failure surface. The total length of the bonded portion of the tie-backs would require calculation of the minimum anchor forces needed and is beyond the scope of the current investigation. The unbonded length of the tie-backs would likely be on the order of 20 feet to 30 feet, based on our interpretation of the slide geometry in relation to the expected wall location.

Conclusions and Opinion of Cost

Conclusions

Attempts to permanently stabilize the upper portion of the coastal bluff edge will prove challenging in light of the scale and magnitude of the forces driving the downslope movements. Under similar circumstances and in a similar setting, we are of the opinion that avoidance (relocating a structure) would be the most practical, economical, and most long-lasting solution. However, we understand the desire of stakeholders to explore the possibility of stabilizing the top of the bluff in order to maintain the current Lighthouse location. That being said, the use of driven spiral nails or drilled and grouted soil nails to stabilize the upper 20 or 30 feet of the bluff face will do little, if anything, to mitigate slope failure within the lower terrace sediments or the larger landscape-scale earthflow movements at depth. We expect that in time, the upper stabilized block may begin to fail as the underlying unreinforced sediments continue to move downslope. Constructing a soil reinforcement system using spiral soil nails or drilled and grouted soil nails, however, may extend the life of the lighthouse at its current location into the near future and provide stakeholders additional time in planning for a more permanent location for the lighthouse.

The more permanent solution to stabilizing the coastal bluff edge is the construction of a soldier pile and tie-back wall, but at considerably greater cost. In our opinion, a wall of this type would serve to protect Edwards Street from being impacted by future landsliding. However, constructing a retaining system of this type at the location of the lighthouse would likely be impractical due to site logistics (such as, limited access and working areas).

Construction Costs

Costs for constructing the types of soil reinforcement and restraining systems described above vary considerably. Unit costs for construction were determined from SHN's past and recent project experience, and research of slope stabilization projects elsewhere in northern California. The estimated unit costs are for construction only—they do not include project costs associated with engineering design; the need for additional drilling investigations in the case of a soldier pile and tie-back wall; or costs associated with environmental studies and regulatory agency permitting, and construction management and inspection.

Driven Spiral Soil Nails. For a spiral nail reinforcement system, we estimate that a unit cost of $40 to $75 per square-foot of protected slope face should be anticipated for planning purposes. At a minimum, we anticipate the need to reinforce approximately 2,400 square-feet of slope face at an estimated cost of about $100,000 to $180,000. The advantage of using this type of reinforcement system is that the materials are readily available locally and it could be constructed by a local contractor.

Drilled and Grouted Soil Nails. We estimate a unit cost of $60 to $100 per square-foot for a drilled and grouted soil nail reinforcement system. Specialized drilling equipment will be required to be transported from out of the area. Limitations associated with site access, staging, and working areas could contribute significantly more to the cost. Similar to the spiral soil nail system, we anticipate the need to reinforce approximately 2,400 square-feet of slope face at an estimated cost of about $144,000 to $240,000.

Soldier Pile and Tie-back Wall. Estimating the cost for a soldier pile and tie-back wall is more uncertain. Large, specialized drilling equipment will need to be transported from out of the area. Limitations associated with site access, staging, and working areas could contribute significantly more to the cost. Extensive grading and temporary access roads would be required to be built along the edge of Edwards Street to facilitate the heavy equipment needed for construction. All construction grading would need to be repaired at the close of the project. We estimate a linear foot unit cost of $3,500 to $6,000 and a wall length of about 160 feet. Construction costs are, therefore, estimated to be in the range of about $560,000 to $1,000,000.

Recommendation

Based on the site conditions, the likelihood for additional significant landslide movement in the coming years, and the costs and limitations of the various repair options described above, it is our recommendation that the City and Civic Club pursue a short-term plan to relocate the lighthouse. We strongly recommend some action be taken prior to the next rainy season. It may not be feasible to complete any slope reinforcement in that timeframe due to the permitting constraints. Even if reinforcement could be achieved in the short term, it does not appear feasible to reinforce the slope adequately using driven or drilled soil nails, because it will only be possible to treat the upper slope face. Lower portions of the slope that would be important to reinforce are beyond the reach of equipment that would be used to construct this reinforcement system. Therefore, even if the upper slope is reinforced, significant potential for additional movement of the lower slope would remain, which would continue to threaten the lighthouse. Because ground very near the current site is associated with significantly lower slope instability potential, it appears feasible to relocate the lighthouse to a nearby site and significantly reduce its exposure to landsliding.

Protection of Edwards Street, a main transportation artery in the town of Trinidad, is of vital importance, because we expect the landsliding to encroach into the roadway in the near future. There is a small buffer remaining between the head of the slide and the edge of the roadway, so the risk factor here is not as high as at the lighthouse; however, we would expect encroachment to occur within the next few years. Therefore, we recommend that the City pursue funding to construct a soldier pile wall to isolate the landslide from the road.

Please feel free to call me at 707-441-8855 if you have any questions.

Sincerely,

SHN Engineers & Geologists

PROFESSIONAL GEOLOGIST
GIOVANNI ANTHONY VADURRO
7/20/17
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CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Giovanni A. Vadurro, CEG
Engineering Geologist

GAV:lms

Attachments: 1. Boring Logs
2. Survey Point File

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1

Boring Logs

SHN Consulting Engineers & Geologists, Inc.

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PROJECT: Trinidad Memorial Lighthouse
LOCATION: S. edge Edwards Street
GROUND SURFACE ELEVATION: 159 Feet (NAVD88)
EXCAVATION METHOD: 6-5/8" RHSA (Taber Drilling)
LOGGED BY: G. Simpson

JOB NUMBER: 017052
DATE DRILLED: 5/3/17
TOTAL DEPTH OF BORING: 50 feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER: 35 Feet

BORING NUMBER TML-1

DEPTH (FT)	BULK SAMPLES	SS SAMPLES	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (tsf) by P.P.	% Passing 200	REMARKS
0.0						A/C paving.						
-2.5						Aggregate base rock (FILL).						Pleistocene marine terrace deposits.
-5.0			3 3 4	SM		Poorly-graded SAND and gravel fill based on cuttings.						
-7.5						SILTY SAND, loose, moist, yellowish brown, weak cementation, low dry strength, faint mottles.						
-10.0			5 5 7	SP/ SM								
-12.5						Poorly-graded SAND with SILT, medium dense, moist, brownish yellow (10YR 6/4), weak cementation, low dry strength, contains cemented concretions.						
-15.0			5 7 8	SP								
-17.5						Grades to Poorly-graded SAND, medium dense, gray.						
-20.0			5 7 8	SW		Becomes Well-graded SAND with fine rounded gravel.						
-22.5												
-25.0			6 6 9	SP		Poorly-graded SAND, medium dense, moist, grayish brown (10YR 5/2), weak cementation, low dry strength, predominantly quartz sand.						
-27.5												
-30.0			6 10 14			Contains coarse rounded gravel.						
-32.5												
-35.0 ▽			2 5 23			No recovery.						Groundwater encountered at 35'.
-37.5												
-40.0						Heaving sands; no sampling attempted; drilled to 45'.						Increased drill resistance according to driller.
-42.5						GRAYWACKE SANDSTONE, very dense, dark gray, strong field strength, fresh.						
-45.0			44/5"									Franciscan Complex bedrock.
-47.5						Boring completed to 50'; constructed 2.75" O.D. slope inclinometer to 50'.						
-50.0												

BORING LOG

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Trinidad Memorial Lighthouse
LOCATION: Downslope of main head scarp
GROUND SURFACE ELEVATION: 156 Feet (NAVD88)
EXCAVATION METHOD: 6-5/8" RHSA (Taber Drilling)
LOGGED BY: G. Simpson

JOB NUMBER: 017052
DATE DRILLED: 5/3/17
TOTAL DEPTH OF BORING: 51 feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER: 42 Feet

BORING NUMBER TML-2

DEPTH (FT)	BULK SAMPLES	SS SAMPLES	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (tsf) by P.P.	% Passing 200	REMARKS
0.0				SP		Poorly-graded SAND based on cuttings.						
-17.5						Becomes gravelly; increase in drill resistance.						
-25.0		⊞	3 4 5	SP		Poorly-graded SAND, loose, moist, grayish brown (10YR 5/2), weak cementation, low dry strength, well rounded coarse sand and fine gravel, thinly bedded.						Sampled 25-26.5' interval only.
-32.5						Contains about 30% fine to coarse rounded gravel.						
-40.0						Sand fraction becomes coarse; increase in drill resistance.						Groundwater encountered at 42'.
-43.0						GRAYWACKE SANDSTONE, dense, very hard drilling.						Drill rig chattering.
-47.5						Boring completed to 51'; constructed 2.75" O.D. slope inclinometer to 50'.						Franciscan Complex bedrock.

Depth scale: 0.0, -2.5, -5.0, -7.5, -10.0, -12.5, -15.0, -17.5, -20.0, -22.5, -25.0, -27.5, -30.0, -32.5, -35.0, -37.5, -40.0, -42.5, -45.0, -47.5, -50.0, -52.5, -55.0

BORING LOG

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Trinidad Memorial Lighthouse
LOCATION: SW corner of Lighthouse
GROUND SURFACE ELEVATION: 152 Feet (NAVD88)
EXCAVATION METHOD: 4" Flight Auger (Taber Drilling)
LOGGED BY: A. Call

JOB NUMBER: 017052
DATE DRILLED: 5/4/17
TOTAL DEPTH OF BORING: 34.5 feet
SAMPLER TYPE: SPT
DEPTH TO GROUNDWATER: 33 Feet

BORING NUMBER TML-3

DEPTH (FT)	BULK SAMPLES	SS SAMPLES	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (tsf) by P.P.	% Passing 200	REMARKS
0.0				SP/SM		Poorly-graded SAND with SILT, loose, moist, yellowish brown, mostly fine sand (description from cuttings). Occasional fine well rounded gravel.						Pleistocene marine terrace deposits.
-5.0		X	3 3 3			Poorly-graded SAND with SILT, loose, moist, weak cementation, fine well rounded gravel; thin interbeds of medium sand and fine gravel consisting of decomposed sandstone.						
-10.0		X	2 2 2	SP		Becomes very loose; decrease in silt content.						
-15.0		X	6 6 7	SP		Poorly-graded SAND, trace SILT, medium dense, moist, yellowish brown, weak cementation; grades to fine sand, yellowish brown, occasional fine gravel.						
-20.0		X	4 6 5									
-25.0		X	4 6 7	SW		Becomes well-graded fine to medium SAND.						
-28.0												Increased drill resistance according to driller.
-30.0		X	5 7 8			Sand fraction grades coarser, well-graded, medium dense, moist, yellowish brown.						
-33.0 (groundwater)						Becomes wet at 33'.						Groundwater encountered at 33'.
-34.5						Borehole completed to 34.5 feet; constructed 2.75" O.D. slope inclinometer to 33.5'.						

Depth scale: 0.0, -2.5, -5.0, -7.5, -10.0, -12.5, -15.0, -17.5, -20.0, -22.5, -25.0, -27.5, -30.0, -32.5, -35.0, -37.5, -40.0

BORING LOG

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Trinidad Memorial Lighthouse
LOCATION: SE corner of Lighthouse
GROUND SURFACE ELEVATION: 151 Feet (NAVD88)
EXCAVATION METHOD: 4" Flight Auger (Taber Drilling)
LOGGED BY: A. Call

JOB NUMBER: 017052
DATE DRILLED: 5/4/17
TOTAL DEPTH OF BORING: 35 feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER: not encountered

BORING NUMBER TML-4

DEPTH (FT)	BULK SAMPLES	SS SAMPLES	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (tsf) by P.P.	% Passing 200	REMARKS
0.0				SP/SM		Poorly-graded SAND with SILT, loose, moist, yellowish brown, mostly fine sand (description from cuttings).						Pleistocene marine terrace deposits.
-5.0			5 9 10									
-10.0			5 6 8	SP		Poorly-graded SAND with GRAVEL, medium dense, moist, trace silt; well rounded fine gravel.						
-15.0			2 3 6	SM		SILTY SAND, loose, moist, yellowish brown.						Increased drill resistance according to driller.
-20.0			4 6 7	SP		Poorly-graded SAND, medium dense, moist, weak cementation, fine sand.						
-25.0			7 7 10			Interbedded medium to coarse Sand to Silty Sand.						
-30.0			5 7 8	SW		Well-graded SAND, medium dense, moist, yellowish brown, fine gravel.						
-33.0						Drill rig chatter; increased drill resistance.						
-35.0						Borehole completed to 35 feet; constructed 1-1/2" piezometer to 33'; screened 13-33'.						

Depth scale: 0.0, -2.5, -5.0, -7.5, -10.0, -12.5, -15.0, -17.5, -20.0, -22.5, -25.0, -27.5, -30.0, -32.5, -35.0, -37.5, -40.0

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

BORING LOG

2

Survey Point File

Trinidad Memorial Landslide Survey Point File- SHN Project Number 017052.100

Point #	X-coord. (ft)	Y-coord. (ft)	Elevation (ft)	Point Description
101	2276082.812	5970594.124	154.668	CP PK BY OTHERS
210	2276091.943	5970734.882	163.142	MON CL
212	2276095.951	5970438.439	146.424	MON CL 9001
312	2276095.95	5970438.456	146.352	CHK CP 212
500	2276091.296	5970648.177	158.636	I-1 NRIM PUNCH MARK
501	2276090.989	5970648.212	157.94	I-1 TOC N NOTCH
502	2276066.035	5970643.524	155.785	I-2 NRIM PUNCH MARK
503	2276065.737	5970643.436	155.09	I-2 TOC N NOTCH
504	2276033.118	5970753.311	152.51	P-1 TOC
505	2276033.33	5970753.374	152.774	P-1 NRIM PUNCH MARK
506	2276037.824	5970729.355	152.335	I-3 NRIM PUNCH MARK
507	2276037.523	5970729.318	151.921	I-3 TOC N NOTCH
1001	2276074.333	5970600.213	154.679	RC-1
1002	2276063.607	5970630.994	155.802	RC-2
1003	2276075.538	5970653.291	158.168	RC-3
1004	2276070.795	5970674.467	158.65	RC-4
1005	2276055.415	5970641.384	152.787	RC-5
1006	2276066.117	5970703.235	156.337	RC-6
1007	2276040.548	5970681.456	149.094	RC-7
1008	2276042.549	5970705.498	150.33	RC-8
1009	2276027.058	5970762.06	152.044	RC-9
1010	2276044.412	5970766.19	153.104	RC-10
1011	2276019.234	5970739.069	149.729	RC-11
1012	2276050.893	5970723.9	152.801	RC-12
1013	2276027.501	5970712.311	145.912	RC-13
1014	2276052.617	5970659.73	153.647	RC-14
1050	2276049.485	5970753.453	153.086	CNC AP
1051	2276030.356	5970752.144	152.578	CNC AP
1052	2276031.844	5970732.362	152.291	CNC AP
1053	2276047.38	5970733.424	152.939	CNC EDG GB
1054	2276040.099	5970723.776	151.281	SCRP

Explanation

CP = control point
MON CL = centerline monument
RC = slope movement monitoring point
SCRP = landslide headscarp
TOC = top of slope inclinometer casing

1055	2276044.208	5970718.752	152.037	SCRP
1056	2276046.242	5970716.915	151.882	SCRP
1057	2276047.872	5970713.672	151.465	SCRP
1058	2276048.178	5970705.517	150.473	SCRP
1059	2276048.856	5970701.682	150.298	SCRP
1060	2276050.989	5970696.801	150.335	SCRP
1061	2276054.498	5970689.191	151.559	SCRP
1062	2276055.939	5970680.986	151.166	SCRP
1063	2276062.098	5970668.653	154.956	SCRP
1064	2276061.026	5970657.559	153.815	SCRP
1065	2276060.731	5970650.165	154.702	SCRP
1066	2276059.533	5970645.976	154.969	SCRP
1067	2276060.045	5970643.122	155.082	SCRP
1068	2276058.839	5970639.554	154.735	SCRP
1069	2276058.297	5970635.533	154.506	SCRP
1070	2276058.262	5970633.244	154.037	SCRP
1071	2275844.043	5970653.104	41.591	TOE AT SPRING
1072	2275375.884	5970752.272	5.722	BABY SEALION
1073	2276033.379	5970753.383	152.78	CHK P-1 NRIM
1074	2276027.099	5970762.061	152.035	CHK RC-9